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# EFFECTS OF STRAIN RATE IN CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS OF COHESIVE SOILS

## REPORT 1. VICKSBURG SILTY CLAY (CL)

ARMY ENGINEER WATERWAYS EXPERIMENT STATION,
VICKSBURG, MISSISSIPPI

FEBRUARY 1970

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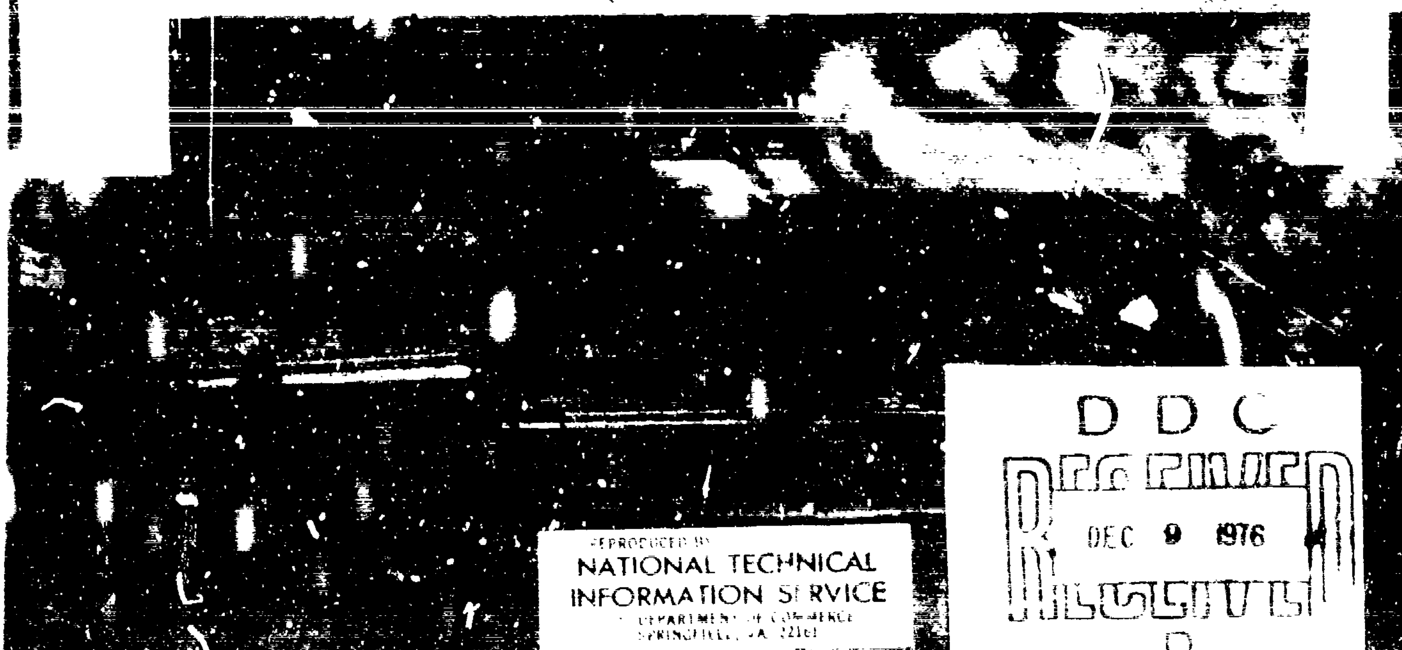
R. F. Esquivel-Diaz, J. R. Compton





February 1976

Sponsored by Office, Chief of Engineers, U. S. Army

Conducted by U. S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

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ARMY-MRC VICKSBURG, MISS.

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## Foreword

This investigation was conducted for the Office, Chief of Engineers (OCE), by the U. S. Army Engineer Waterways Experiment Station (WES) under the Engineering Study Item ES 516, "Evaluation of Laboratory Equipment and Testing Procedures." The testing program was authorized by OCE first indorsement dated 18 September 1967 to WES letter dated 14 July 1967, subject: Rate of Strain in F Triaxial Compression Tests, ES 516.

The study was conducted by Messrs. R. F. Esquivel-Diaz and F. G. A. Hess, Laboratory Research Section, Embankment and Foundation Branch, Soils Division, under the direct supervision of Mr. B. N. MacIver, Chief, Laboratory Research Section, and under the general supervision of Mr. J. R. Compton, Chief, Embankment and Foundation Branch, and Messrs. W. J. Turnbull and A. A. Maxwell, Chief and Assistant Chief, respectively, Soils Division. This report was prepared by Messrs. Esquivel-Diaz and Compton.

COL Levi A. Brown, CE, was Director of WES during preparation and publication of this report. Mr. F. R. Brown was Technical Director.

## Contents

## Conversion Factors, British to Metric Units of Measurement

British units of measurement used in this report can be converted to metric units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| inches | 25.4 | millimeters |
| pounds | 0.45359237 | kilograms |
| pounds per square inch | 0.070307 | kilograms (force) per square centimeter |
| | 6.894757 | kilonewtons per square meter |
| tons per square foot | 0.9764855 | kilograms (force) per square centimeter |
| | 95.76052 | kilonewtons per square meter |
| pounds per cubic foot | 16.0185 | kilograms per cubic meter |

## Summary

The results of consolidated-undrained (termed R test in Corps of Engineers nomenclature) triaxial compression tests with pore pressure measurements performed on Vicksburg silty clay (CL) are presented and analyzed in this report. All triaxial specimens were compacted with a Harvard miniature compactor to 95 percent of standard maximum density with water contents 2 percentage points wet of standard optimum. After back-pressure saturation and consolidation under four different chamber pressures, the specimens were axially loaded at rates of strain varying from 0.001 to 1.0 percent/min. The purpose of the tests was to evaluate the effects, if any, of different rates of strain on the shear strength and deformation characteristics of this particular soil. Data presented include pore pressure observations, magnitudes of deviator stresses, Mohr's diagrams, and stress path plots.

R triaxial test results indicate that this lean clay, which has a liquid limit of 34, plastic limit of 22, and plasticity index of 12, is relatively insensitive to the rates of strain used in axial loading. When other materials have been tested at different rates of strain in succeeding phases of the program, more definitive guidance on rates of strain for various fine-grained soils should be possible.

# EFFECTS OF STRAIN RATE IN CONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TESTS OF COHESIVE SOILS

VICKSBURG SILTY CLAY (CL)

## Introduction

1. The present practice of the Corps of Engineers, as set forth in EM 1110-2-1906,* is to perform the consolidated-undrained triaxial compression test, termed R test in Corps of Engineers nomenclature, by first completely saturating each of at least three identical soil specimens and isotropically consolidating each specimen under a different effective pressure. Then drainage connections to the specimen are closed and the specimen is compressed to at least 15 percent axial strain. When it is desired to determine only total stresses from the test, pore water pressures developed during shear are not measured in routine testing.

2. Since R triaxial tests are time-consuming and expensive, it is highly desirable that procedures used by the Division laboratories be as economical as possible. An important element of the testing procedure is the time to failure or duration of the axial loading phase to the maximum deviator stress. Currently, the time to failure is specified to be between 60 and 120 min for cohesive soil. For tests in which it is desired to develop stress-strain curves to 15 percent strain and where maximum deviator stress is reached at low strains, the present procedure is time-consuming. For example, if a constant strain rate is used during shear and maximum deviator stress occurs at 3 percent strain, some clay samples are sheared for 120 min to reach this peak stress. If the test is carried to 15 percent strain, about 8 hr more would be required to complete the test. The purpose of this investigation is to determine the rate of strain in the R test that will give the lowest value for maximum deviator stress so that the most conservative total stress envelope can be developed.

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* Department of the Army, Office, Chief of Engineers, "Engineering and Design: Laboratory Soils Testing," Engineer Manual EM 1110-2-1906, 10 May 1965, Washington, D. C.

3. R triaxial compression tests were performed by the U. S. Army Engineer Waterways Experiment Station (WES) soils laboratory on specimens of Vicksburg silty clay (CL) standard soil.* Average Atterberg limits of the material based on previous standard soil sample tests were: liquid limit, 34; plastic limit, 22; plasticity index, 12.** Average specific gravity was 2.68; percent finer by weight than 2 microns averaged 18. Gradation curves are presented in fig. 1.

4. Assembly and calibration of equipment were begun in October 1967, and the test program was accomplished during the period December 1967 through April 1968. Specimens were 1.4 in.† in diameter by 3 in. high, and were compacted by the Harvard miniature compaction procedure to 95 percent of standard maximum dry density at 2 percent above standard optimum water content. Specimens were fully saturated by back pressure and consolidated under pressures of 0.5, 1.5, 3.0, and 5.0 tsf. Under each consolidation pressure, specimens were sheared undrained at rates of strain of 1.0, 0.5, 0.1, 0.01, and 0.001 percent/min until a maximum axial strain of 20 percent was reached. Pore water pressures were measured during shear. After shear, each specimen was cut horizontally into seven slices to determine the distribution of the final water content.

## Description of Equipment

5. A schematic diagram of the testing apparatus is shown in fig. 2. The triaxial chambers with specimen bases and caps and axial loading pistons were those used regularly in the WES Soils Laboratory for Division soil testing.†† The confining pressure was applied by using water as the chamber fluid, pressurized with compressed air, and measured with

* At this writing, a similar testing program is being initiated on standard CH soil sample (Vicksburg buckshot clay).

** W. E. Strohm, Jr., "Preliminary Analysis of Results of Division Laboratory Tests on Standard Soil Samples," Miscellaneous Paper No. 3-813, Apr 1966, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

† A table of factors for converting British units of measurement to metric units is presented on page ix.

†† Op. cit., EM 1110-2-1906, Appendix X, Fig. 2.

calibrated Bourdon gages having a capacity of 200 psig. The air pressure was controlled with pressure regulators.

6. Back pressure for saturation was applied with compressed air acting on a column of water inside a calibrated burette. Air pressure was controlled with a pressure regulator and measured with calibrated Bourdon gages. The pore water pressure was measured at the base of the specimen, using Statham pressure transducers with a rated capacity of 250 psi.

7. A triple-unit loading machine (fig. 3) was used to shear the specimens. The load was applied through a worm jack operated by an electric motor with the piston of the triaxial chamber reacting against the load cell. Each of the three load cells, made by Transducers, Inc., had a rated capacity of 200 lb. Rate of strain was controlled with a Zero-Max speed reducer. For the two lowest rates of strain, a Boston V113 worm-gear speed reducer, 20:1, was attached to the Zero-Max speed reducer. Change in height of the specimens during shear was measured with a rectilinear potentiometer (CIC) fixed to one of the loading units.

8. A Mosley Autograf Model 7100A single-point strip chart recorder was used to record the pore water pressure during the saturation phase. Load, vertical displacement, and pore pressure measurements during axial loading were recorded with a Westronics, Inc., Model RM11B strip chart recorder with 24 channels. To operate the recorders, two CEC bridge-balance units, type 8-108, were used with a regulated power supply, Model 40P411FM, from Lambda Electronics Corporation.

## Preparation of Specimens, Testing Procedures, and Test Results

### Compaction

9. The optimum water content and maximum dry density of standard soil sample CL, based on standard compaction tests by various Division laboratories, were 16.6 percent and 109.2 pcf, respectively.* For the rate of strain tests, the desired compaction conditions were: water content 2 percentage points above optimum (18.6 percent) and a density equal to

* Strohm, op. cit.

95 percent of maximum dry density (103.7 pcf); see fig. 4. As shown in table 2, the actual initial compacted density of the test specimens ranged from 103.4 to 104.0 pcf and the water content ranged from 18.2 to 19.0 percent.

10. Sufficient soil to compact a single specimen was mixed with water to attain the desired water content; by trial, it was found that about 0.8 percent more had to be added because of loss of moisture during processing and compacting. After the soil was thoroughly mixed in the humid room, it was placed in an airtight glass jar to cure in the humid room for a minimum period of 24 hr. Compaction was accomplished with a Harvard miniature compactor, using a compaction rod with 1/2-in.-diam bearing surface. The rod was provided with a spring that gave a compaction force of about 12.6 lb when the spring was compressed.

11. All specimens were compacted in the humid room in 10 layers having approximately the same amount of material in each layer. By trial, it was found that the desired dry unit weight could be obtained by using 14 tamps per layer. After the specimen was compacted and found to meet the desired compaction conditions, it was immediately set up in the triaxial chamber using two standard rubber membranes separated by a coat of silicone grease. No filter paper strips were used on the sides of the specimens. The membranes were sealed using two O-rings at each end to fasten them to the base and cap.

Saturation and con-
<u>solidation procedures</u>

12. <u>Specimen setup.</u> In tests R1 through R4, the specimens were allowed to remain overnight with no chamber pressure applied before commencing saturation the following day. Check test R4a in which a small chamber pressure was maintained overnight showed results similar to companion test R4, indicating that no change took place during the period without chamber pressure. All other test specimens were subjected to 2-psi chamber pressure immediately after setting up in the triaxial chamber, and the back-pressure saturation procedure was generally initiated shortly thereafter.

13. <u>Specimen saturation.</u> As indicated by the footnotes in table 1,

there were some variations in the back-pressure saturation procedure used in the various tests, particularly in the initial tests. In all tests except R1 through R4, a chamber pressure of 2 psi was imposed initially, and burette readings were made until equilibrium was reached. Following this, the chamber pressure was increased, generally to 7 psi, with a simultaneous application of 5-psi back pressure. In all subsequent applications of chamber and back-pressure increments, the difference between the chamber pressure and the back pressure was maintained at 2 psi. The most efficient procedure with the least demands on operator surveillance was to allow the initial chamber pressure and back pressure to be maintained on the specimen overnight, following which the chamber and back-pressure increments were applied at intervals of 10 to 15 min. During the saturation process, the response of the pore pressure transducer was recorded. Following back-pressure saturation, valve B (fig. 2) was closed, the vertical dial indicator was read, and the specimen was ready for consolidation under the desired pressure.

14. Specimen consolidation. With valves B and C closed (see fig. 2), the chamber pressure was increased so that the difference between the chamber pressure and back pressure was equal to the desired effective consolidation pressure, and the pore pressure was observed to verify the completeness of the saturation process. Then, valves B and C were opened, and burette and dial indicator readings were made at time intervals. When consolidation was completed, valve B was closed and the specimen was ready for axial loading. It was observed, after consolidating a few specimens under different lateral pressures, that primary consolidation was completed about 2 hr after the consolidation process started, and that the CL soil showed no important secondary consolidation. For this reason, the consolidation phase was ended when 24 hr had elapsed after the beginning of consolidation. Duration of the various test phases is shown in table 1.

## Test results

15. Table 2 summarizes the triaxial compression test data, showing not only the axial loading data but also initial specimen conditions and specimen conditions after back-pressure saturation and consolidation. The tests are grouped in this table according to the chamber pressure used.

The before- and after-test specimen conditions will be discussed first, following which the shear test data will be discussed.

16. Specimen conditions. Initial water contents and compacted dry densities for the tests listed in table 2 had average values of 18.6 percent and 103.7 pcf, respectively, which were exactly those desired. The greatest deviation among all specimens was $\pm 0.4$ percent for the water content and about $\pm 0.4$ pcf for the density. It is to be noted that many specimens were compacted and rejected because their water content and/or density were not close to the desired values. It is also to be noted that the water content of 18.6 percent for the shear test specimens is somewhat dry of the optimum water content for the compaction effort used to obtain the desired density as shown by the Harvard miniature curve in fig. 4. Changes in water contents and densities caused by saturation and consolidation naturally depended upon the magnitude of the consolidation pressure, as shown by the following tabulation:

| Consolidation Pressure $\bar{\sigma}_c$, kg/cm$^2$ | Average Differences Between End of Consolidation and As-Molded Conditions: Water Content $w$, % | Dry Density $\gamma_d$, pcf |
|---|---|---|
| 0.49 | +4.0 | +0.3 |
| 1.46 | +3.5 | +2.1 |
| 2.93 | +2.6 | +3.5 |
| 4.88 | +1.6 | +4.8 |

Following completion of axial loading, specimens were quickly removed, taken to the humid room, and sliced horizontally into seven slices; the top and bottom slices were 0.2 in. thick; the remaining five slices were cut to be of equal thickness. Water contents were then determined on the individual slices. Vertical distribution of water content within a specimen showed a slight tendency for the central portions of the specimens to have slightly higher water contents than the upper and lower portions (see table 3 and fig. 5). It would be expected that if the rate of strain has a significant effect on the R shear strength of a soil, this effect would make itself known by consistent differences in water content distributions within specimens consolidated under the same pressure, but axially loaded at different rates of strain. However, table 3 indicates that there is no pattern of variation of the water content determined after shear with different rates of strain for tests with the same chamber pressures; therefore, from this standpoint, it is indicated that this CL soil is relatively insensitive to the rate of axial strain.

17. It is of interest to compare the water contents determined directly at the end of test with water contents computed for the after-consolidation conditions (using the initial water contents and water content changes observed during saturation and consolidation as indicated by the burette). Fig. 6 shows generally close agreement between water contents computed by the method described in table 3 and the water contents determined at the end of the tests. Plots of volume changes (as indicated by the burette) and of changes in specimen height (as indicated by the dial gage) during consolidation are shown for four tests under different consolidation pressures in fig. 7.

18. Shear strength and pore pressure data. Plots of deviator stress versus axial strain with test data grouped under each of the four confining pressures used in the test program are shown in figs. 8 and 9. Fig. 10 is a plot of maximum deviator stress versus rate of strain. Since deviator stresses did not peak before 20 percent axial strain, the deviator stress at an axial strain of 15 percent is reported as maximum deviator stress as prescribed in EM 1110-2-1906. Thus, fig. 10 also presents maximum deviator stress versus time to 15 percent axial strain. The Mohr's diagrams in figs. 11 and 12, in which test data are grouped by rate of strain, show that for rates of strain from 0.01 to 1.0 percent/min, a shear strength, based on total stresses, of $\phi = 17.5$ deg and $c = 0.40$ kg/cm$^2$ fits all four plots very well. The shear strength for the tests at rate of strain of 0.001 percent/min (fig. 13) is somewhat higher ($\phi = 18$ deg, $c = 0.53$ kg/cm$^2$).

19. The plot of induced pore pressure versus axial strain in fig. 14 shows no discernible effect of rate of strain on the pore pressures induced by axial loading based on measurements taken at the base of the specimen. Plots were made of pore pressure parameter $A = \dfrac{u - u_o}{\sigma_1 - \sigma_3}$ versus axial

strain, and these are presented in figs. 15-18. Fig. 19 shows the ranges of stress paths for the current tests in which the rate of strain was varied from 0.001 to 1.0 percent/min and also shows stress paths of the R tests by the U. S. Army Engineer Division, Southwestern (SWD), and by WES in the 1965 standard soil sample test program on the same material, using rates of strain of 0.06 to 0.07 percent/min.

## Conclusions

20. Results of this test program indicate that this Vicksburg lean clay compacted at a water content 2 percent wet of optimum and to a dry density of 95 percent of standard Proctor maximum dry density is only slightly sensitive to rates of axial strain in the R test. Under effective confining pressures of 0.49, 1.46, 2.93, and 4.88 kg/cm$^2$, rates of strain ranging from 0.5 to 0.01 percent/min gave the lowest values of maximum deviator stress. However, it appears that for this material at this condition, axial loading might be as fast as 1 percent of axial strain per minute without appreciably affecting the total stress results. When other materials have been tested at different rates of strain in succeeding phases of the program, more definitive guidance on rates of strain for various fine-grained soils should be possible.

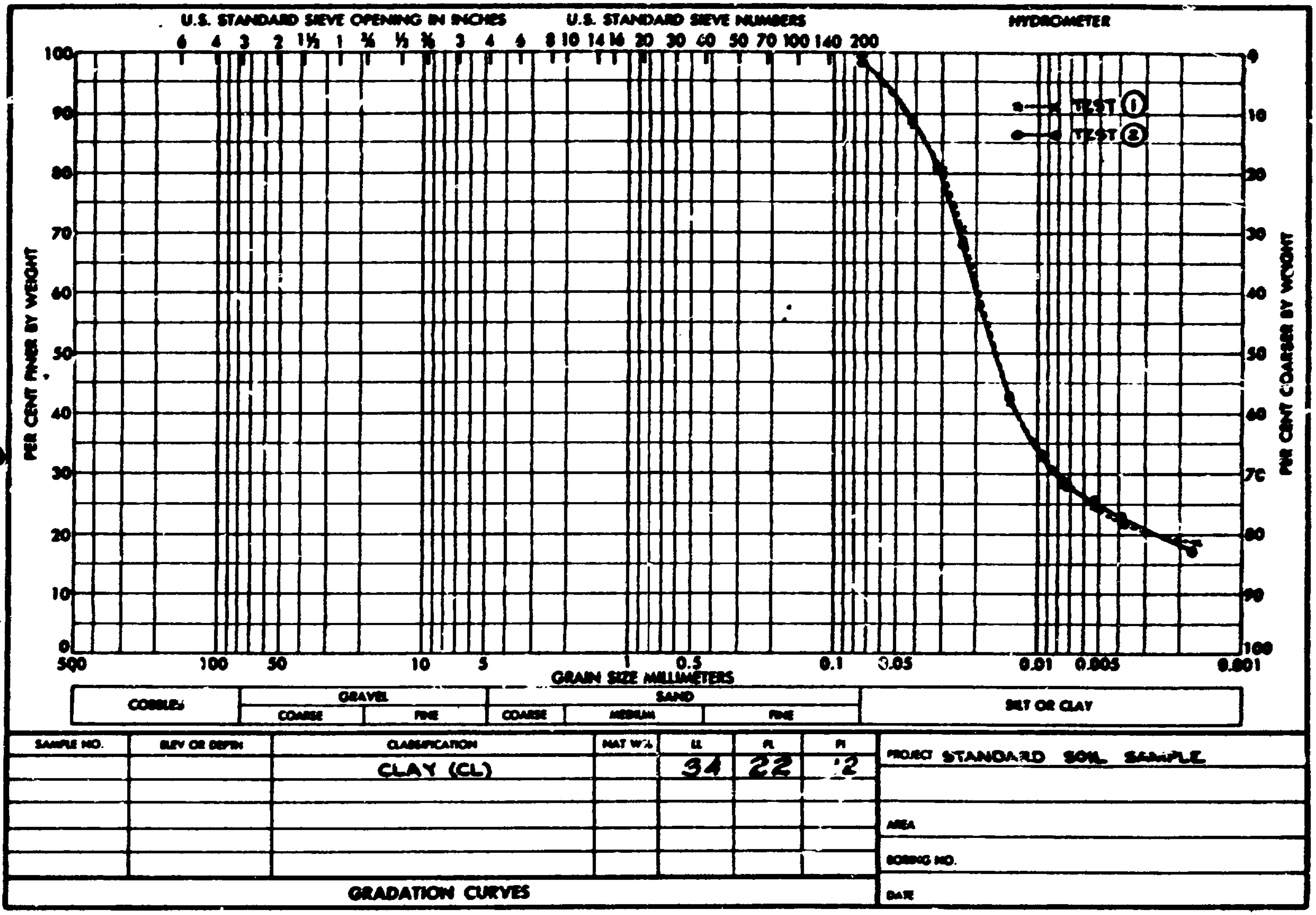



Fig. 1. Gradation curves

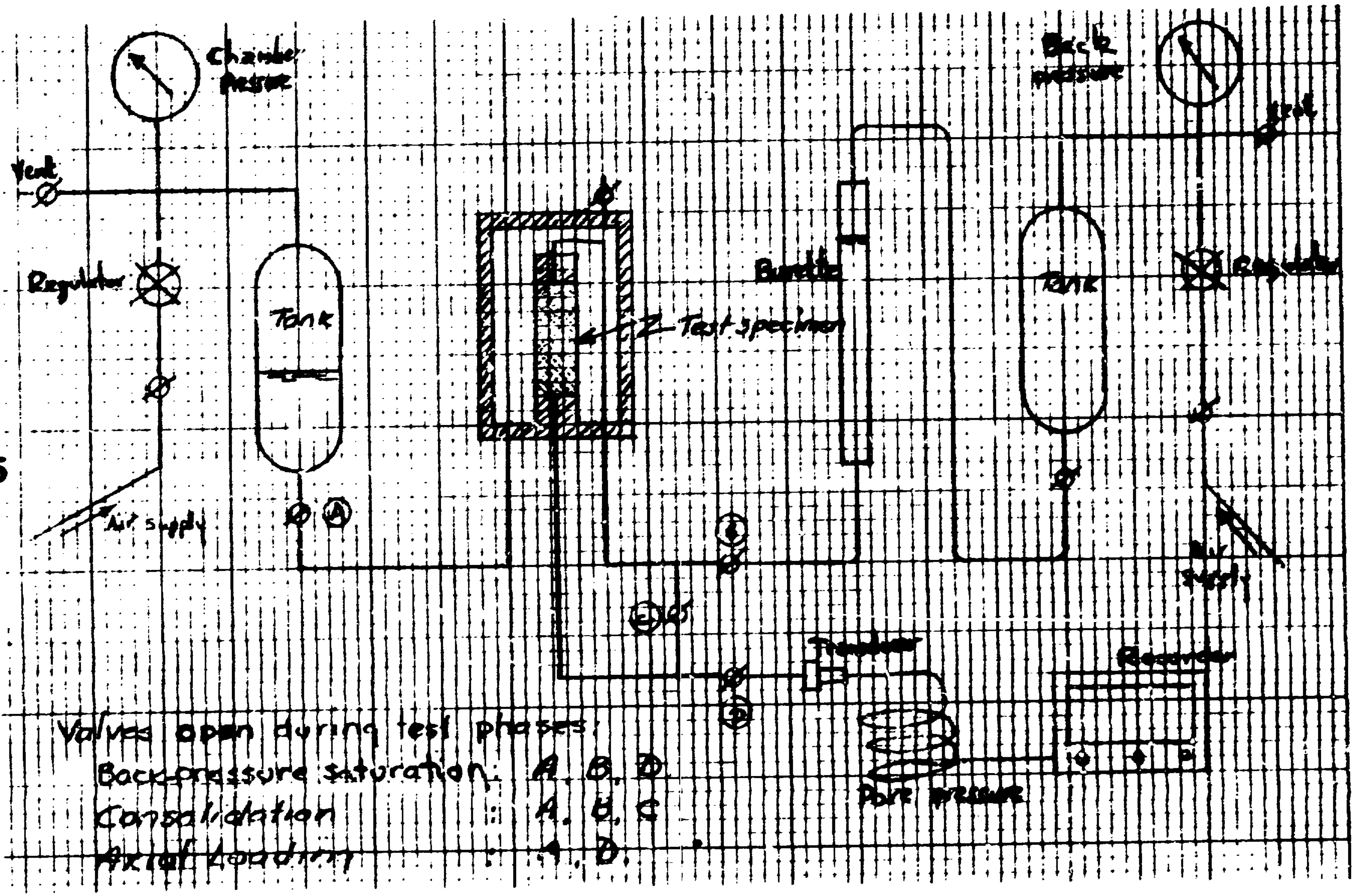



Fig. 2. Schematic diagram of testing apparatus

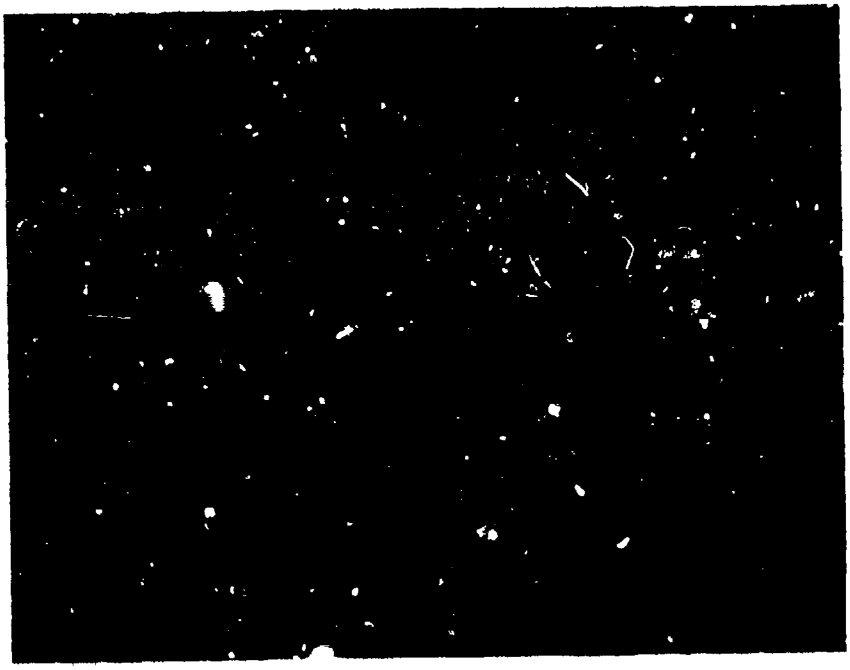


Fig. 3. Triple-unit loading machine

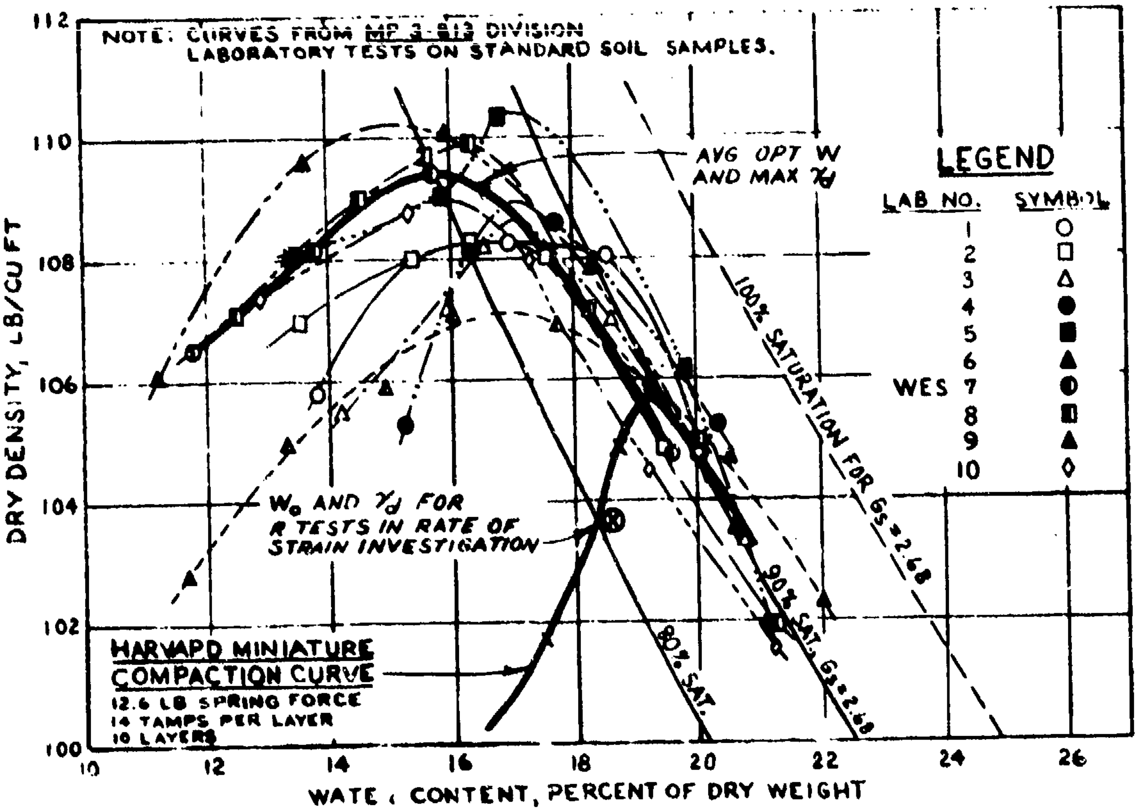



Fig. 4. Standard-effort compaction curves, CL soil

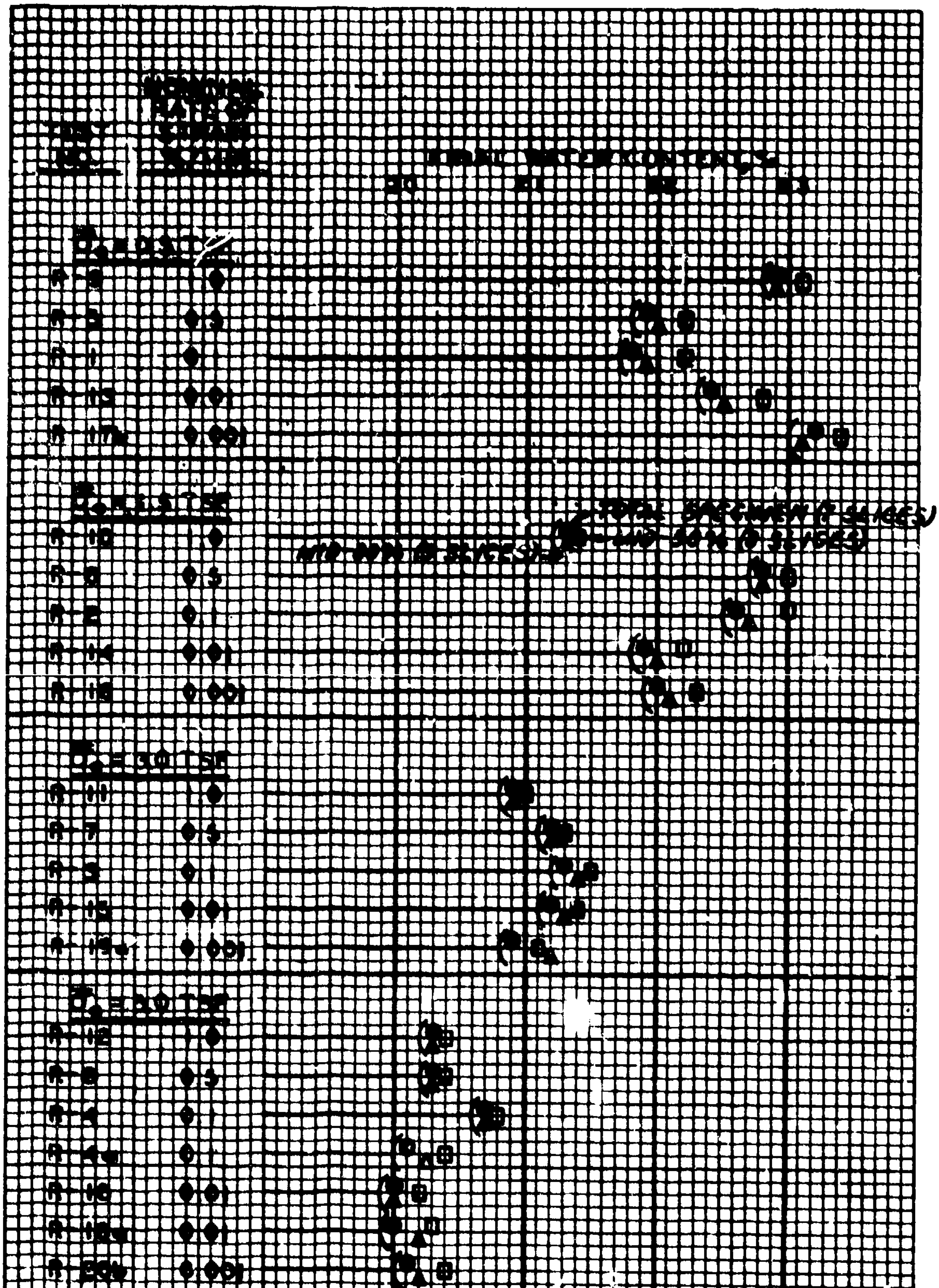


Fig. 5. Water content distribution after test

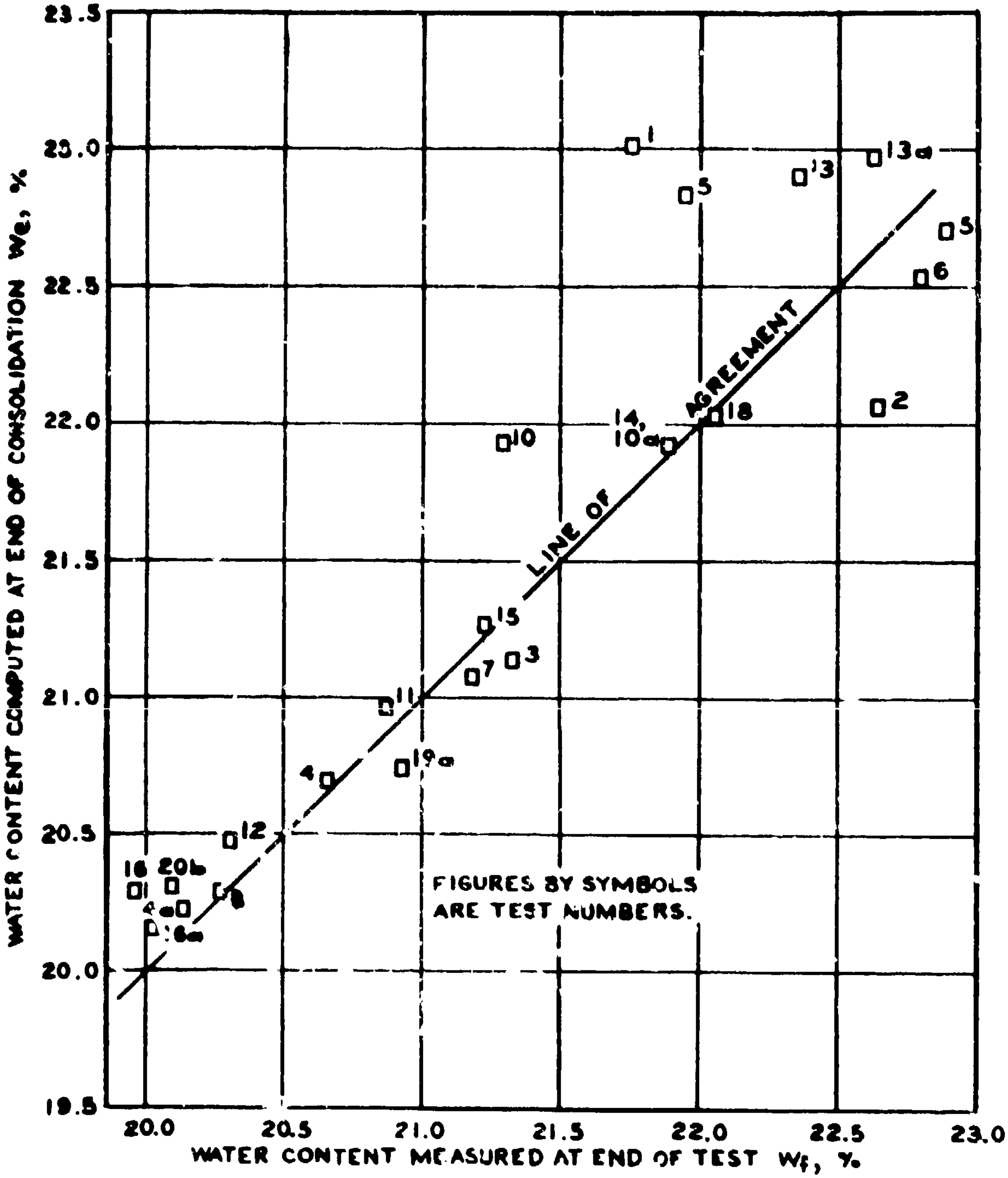



Fig. 6. Water contents computed at end of consolidation versus water contents determined at end of test

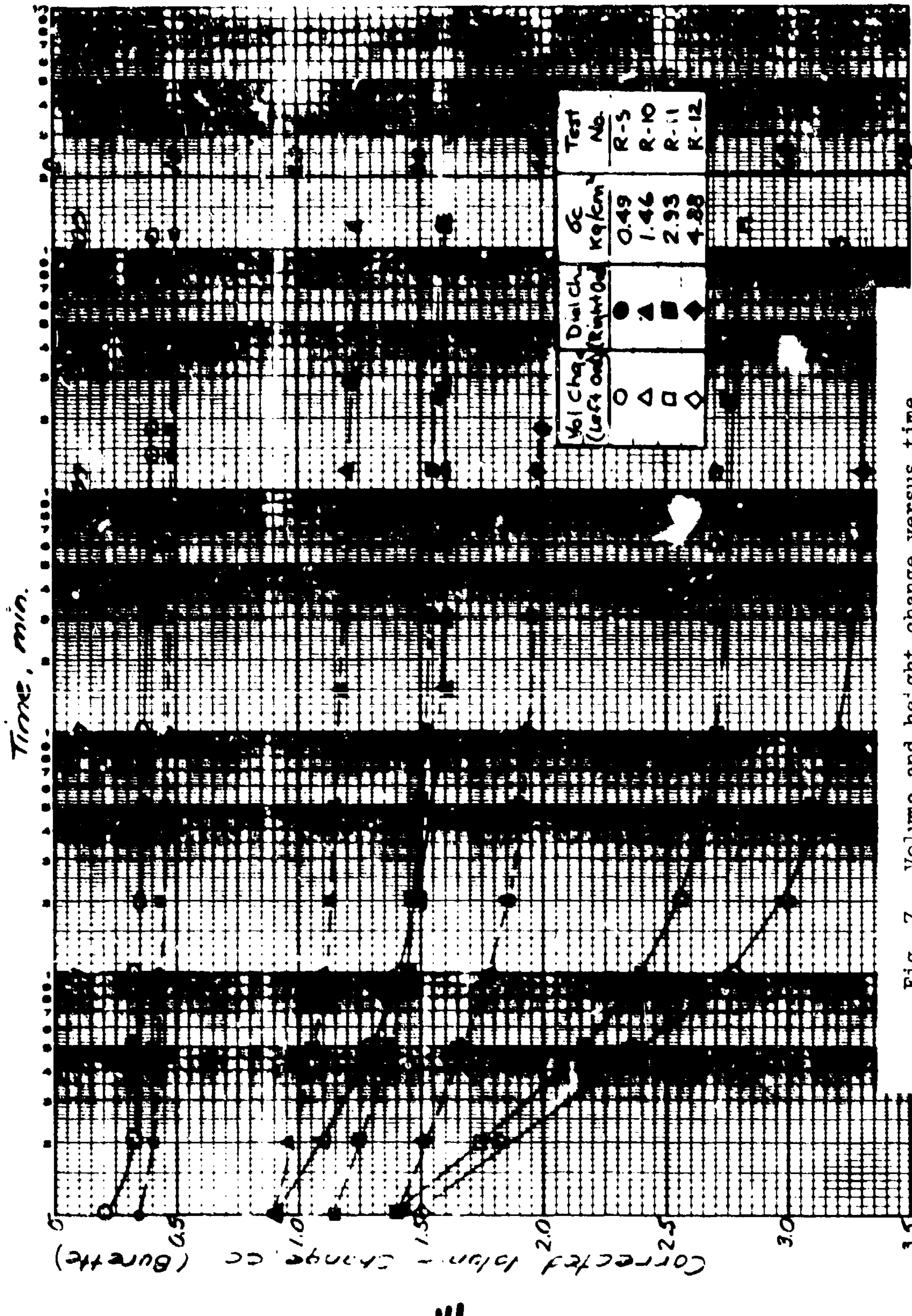
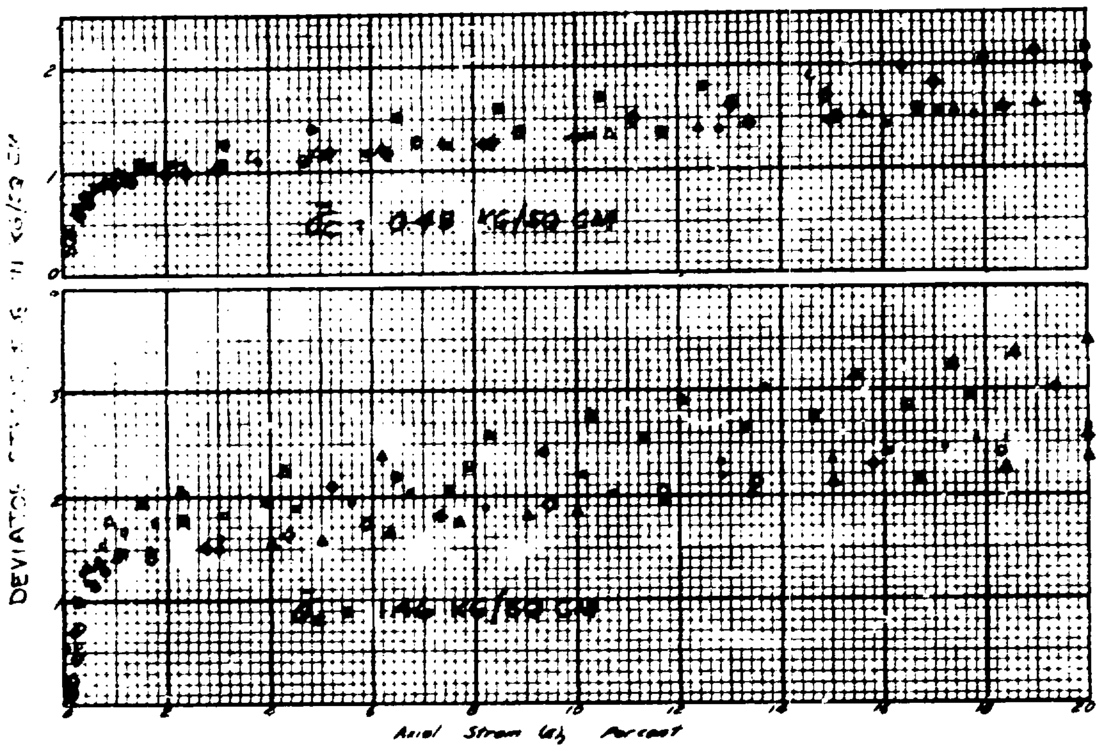



Fig. 7. Volume and height change versus time




| Symbol | Nominal Rate of Strain %/min | Test Numbers $\bar{\sigma}_c$ in kg/sq cm 0.49 | 1.46 |
|---|---|---|---|
| △ | 1.0 | R-9 | R-10 |
| ▲ | 1.0 | | R-10a |
| □ | 0.5 | R-6 | R-6 |
| ○ | 0.1 | R-1 | R-2 |
| ◇ | 0.01 | R-13 | R-14 |
| ◆ | 0.01 | R-13a | |
| ● | 0.001 | R-17b | R-18 |

Fig. 8. Deviator stress versus axial strain. $\bar{\sigma}_c$ = 0.49 and 1.46 kg/cm$^2$

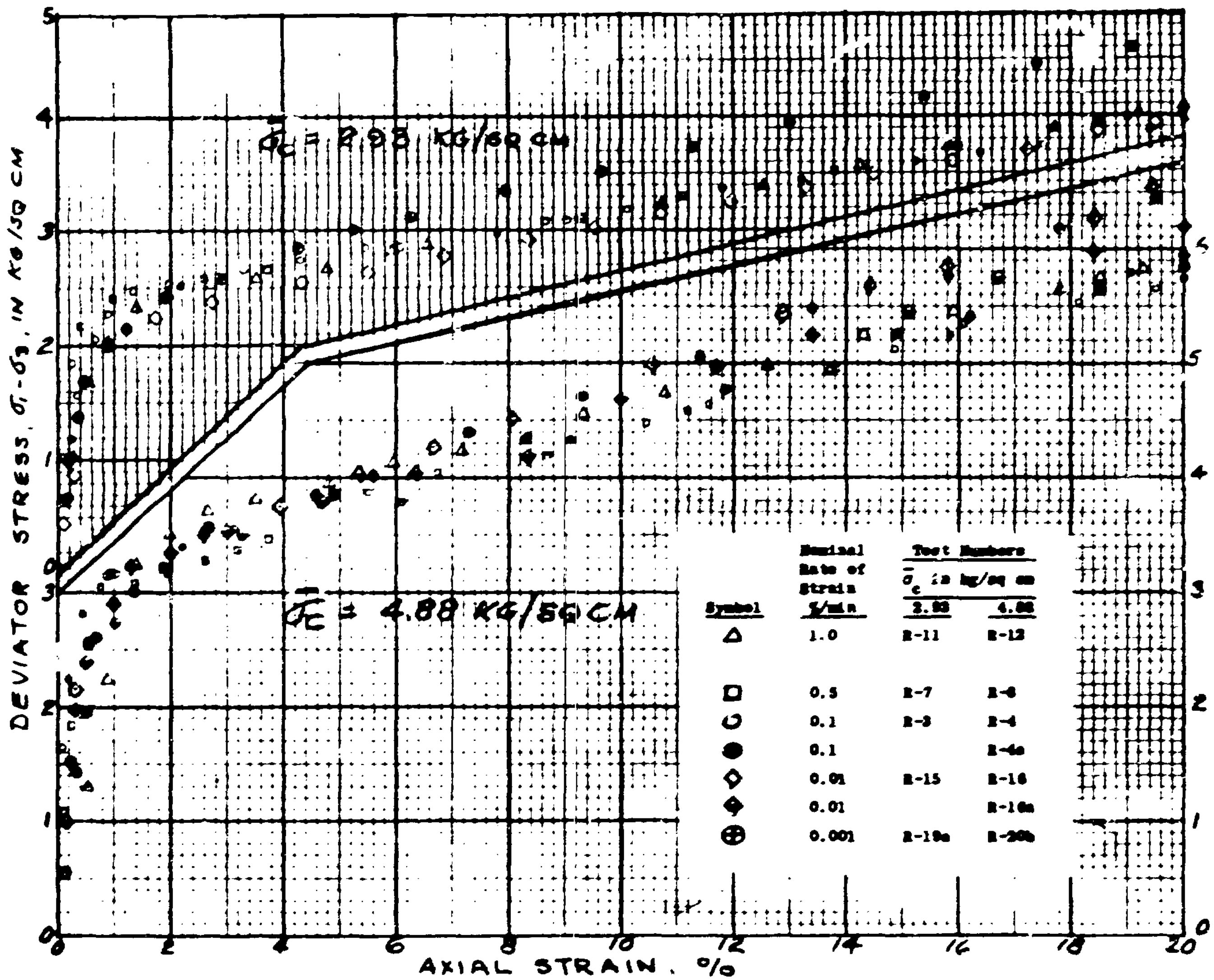



Fig. 9. Deviator stress versus axial strain. $\bar{\sigma}_c$ = 2.93 and 4.88 kg/cm$^2$

Fig. 10. Rate of strain versus maximum $\sigma_1 - \sigma_3$

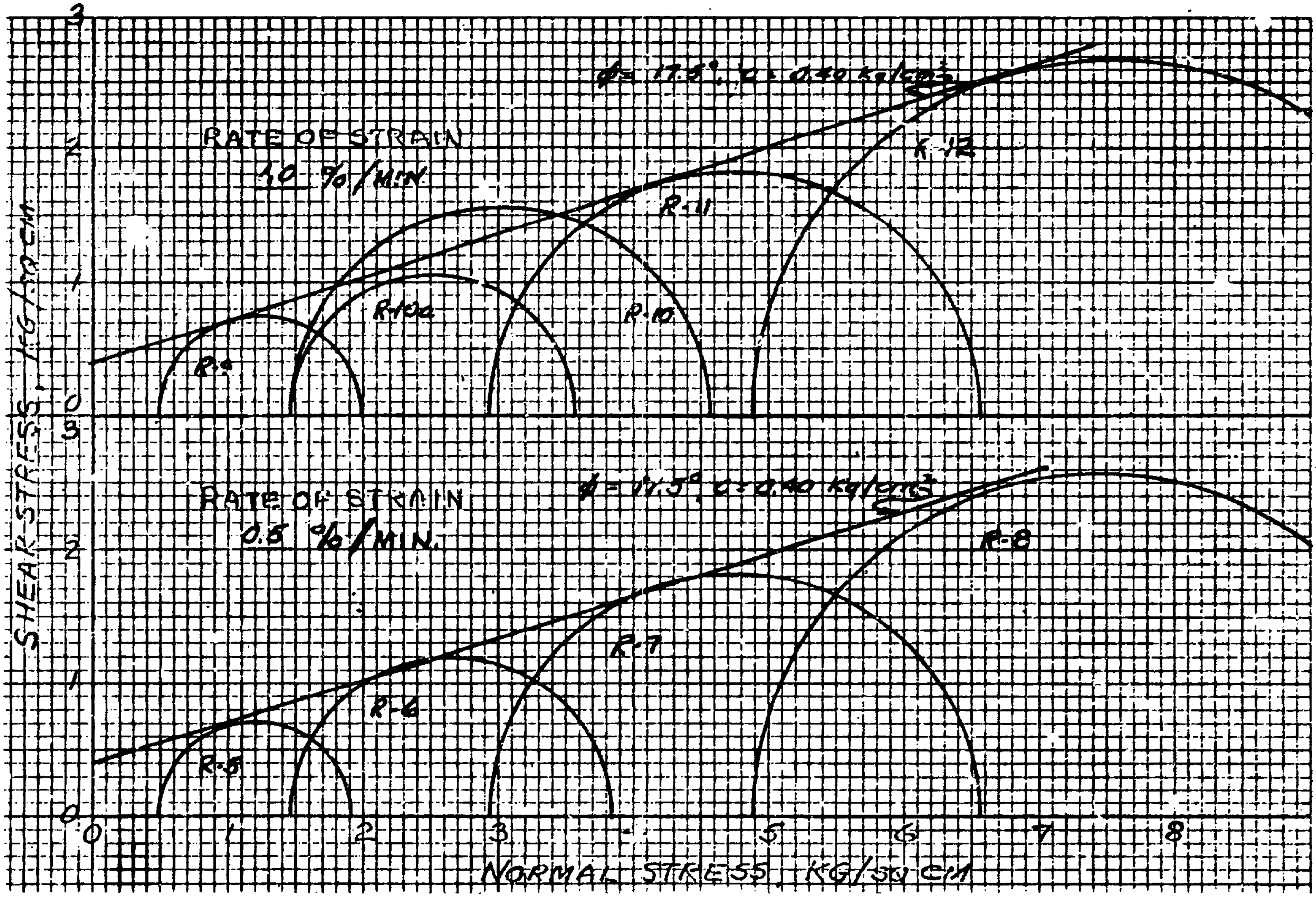



Fig. 11. Mohr's envelopes based on total stresses at 15 percent axial strain. Rate of strain = 1.0 and 0.5%/min

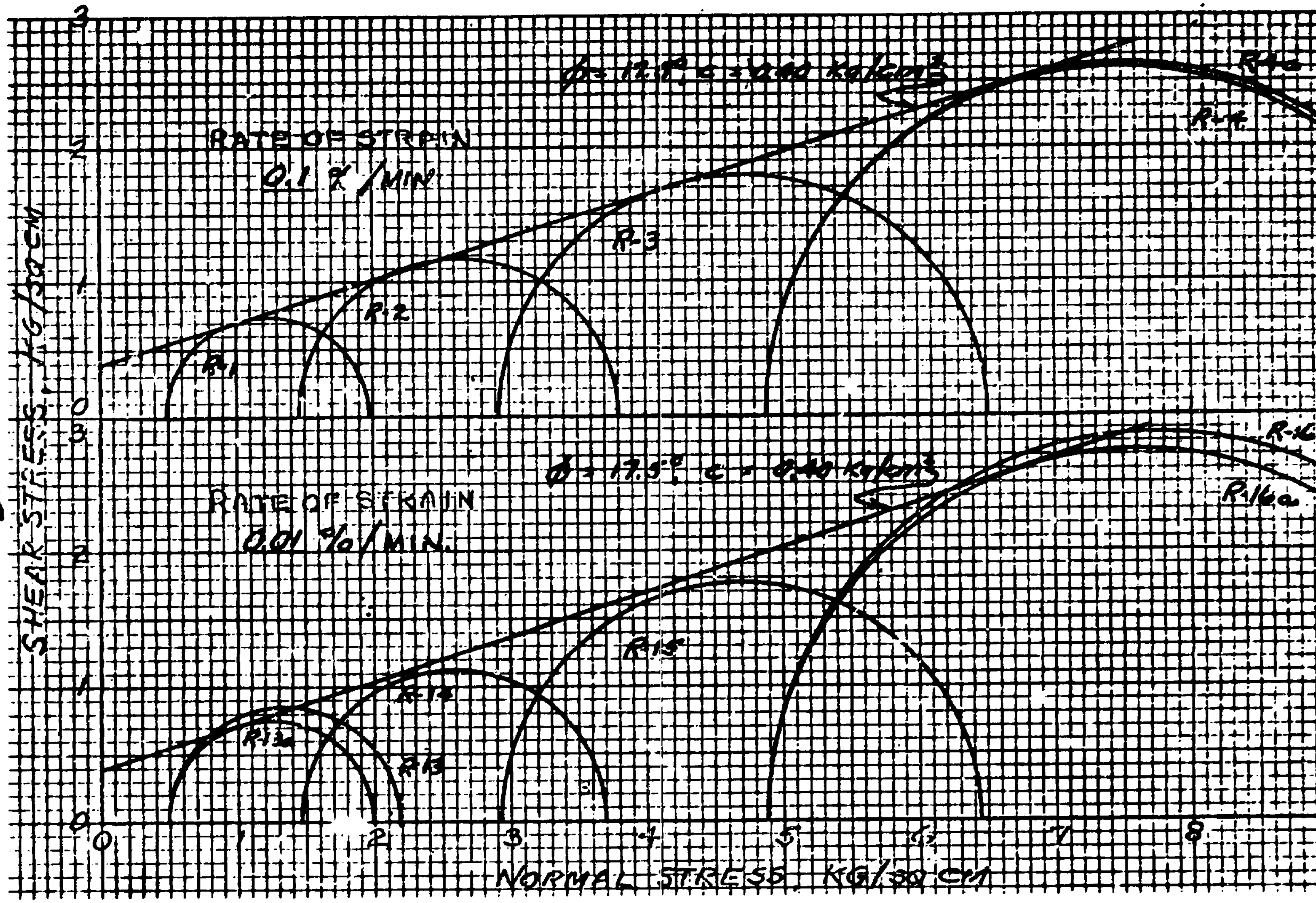



Fig. 12. Mohr's envelopes based on total stresses at 15 percent axial strain. Rate of strain = C.1 and 0.01%/min

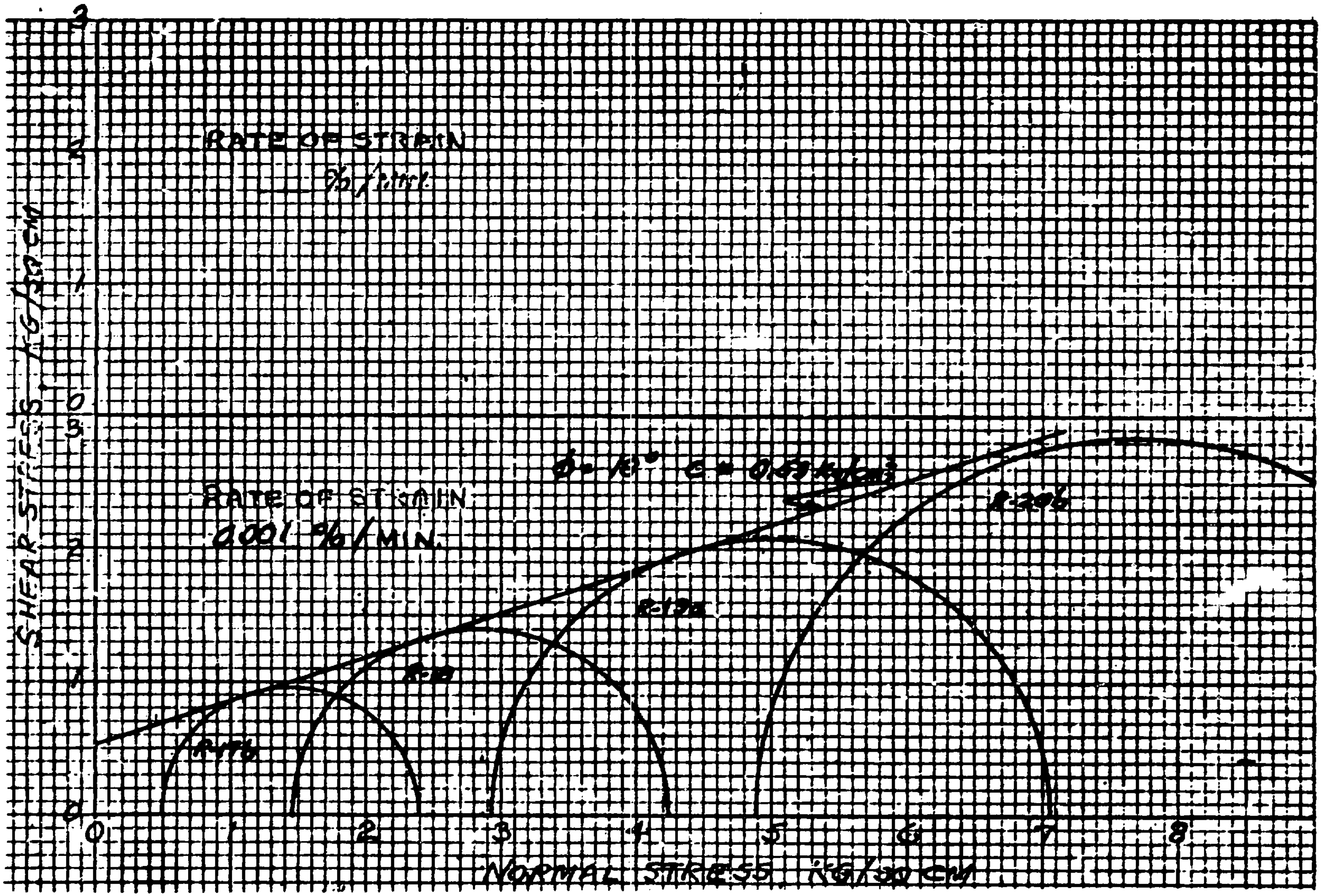



Fig. 13. Mohr's envelopes based on total stresses at 15 percent axial strain. Rate of strain = 0.001%/min

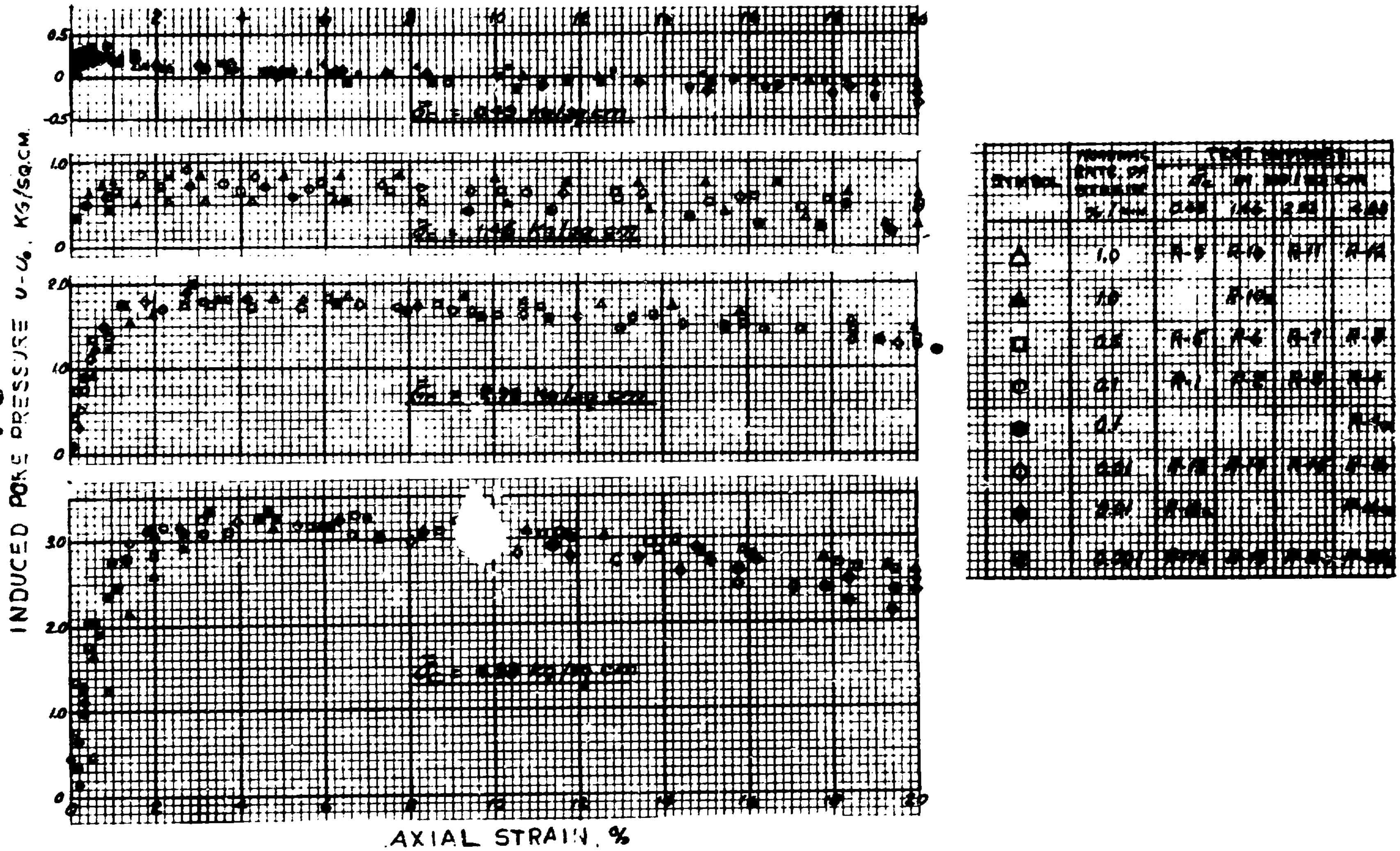



Fig. 14. Induced pore pressure versus axial strain

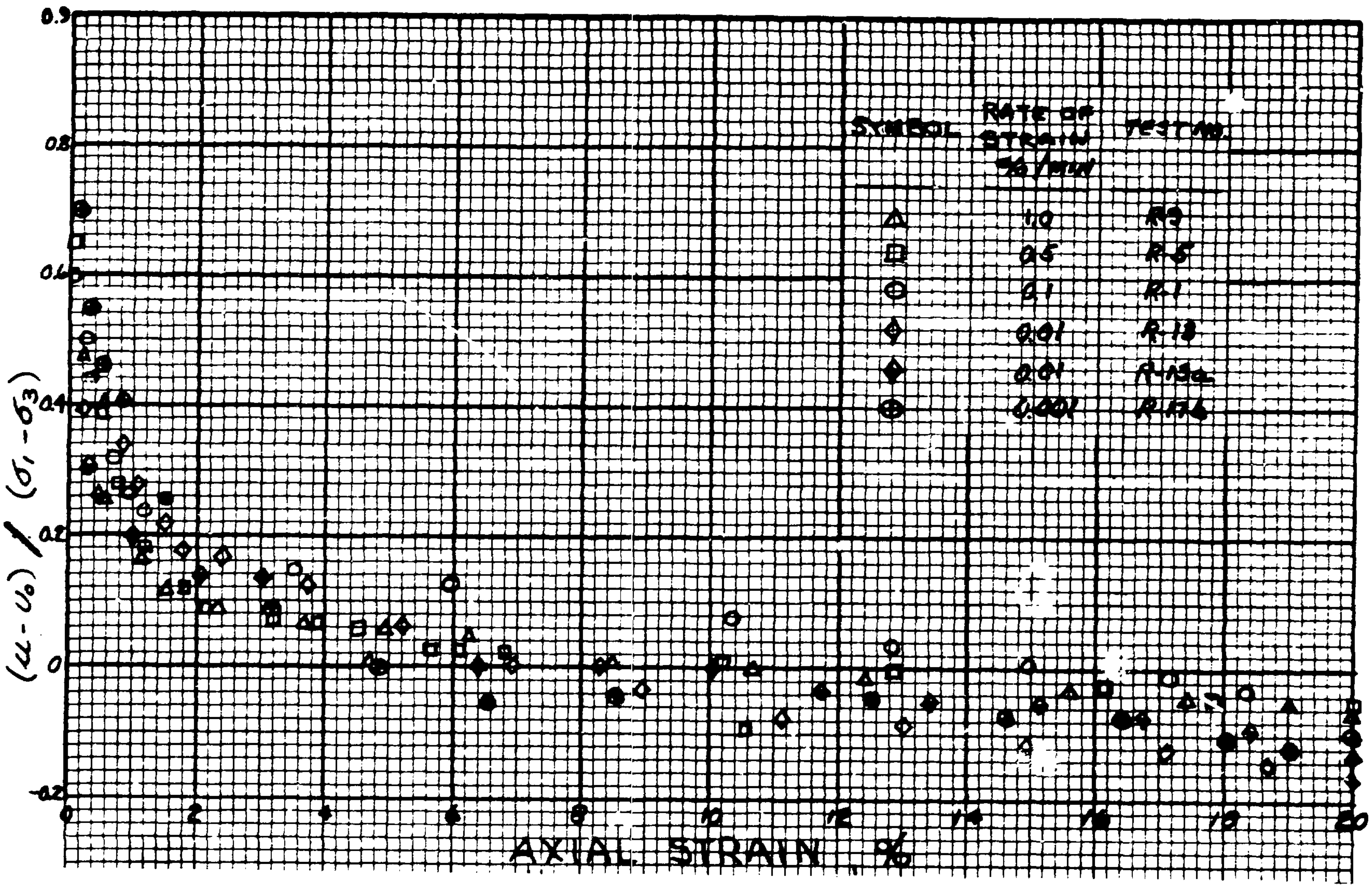



Fig. 15. Pore pressure parameter A versus axial strain. $\bar{\sigma}_c = 0.49$ kg/cm$^2$

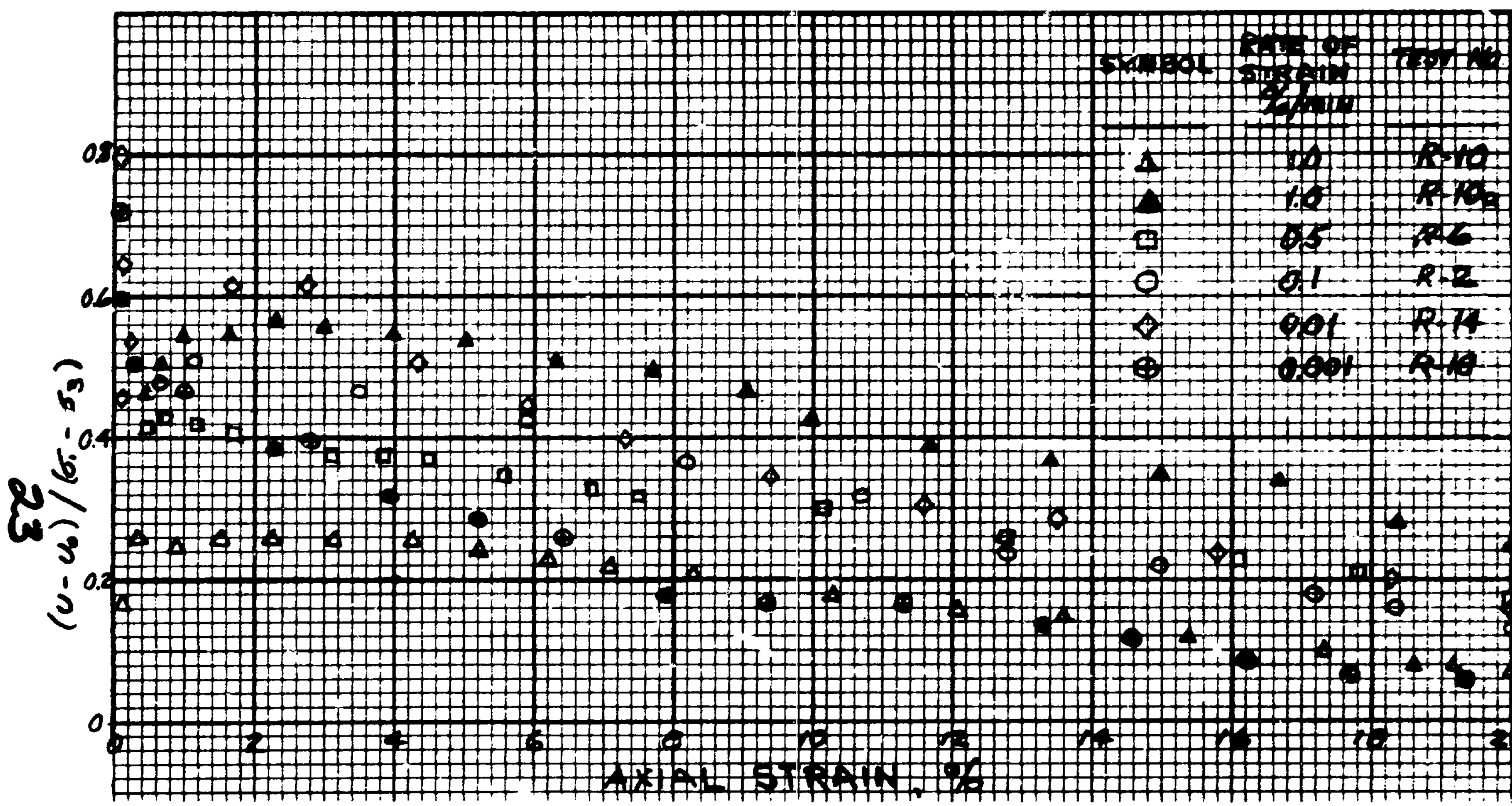



Fig. 16. Pore pressure parameter A versus axial strain. $\bar{\sigma}_c = 1.46\ kg/cm^2$

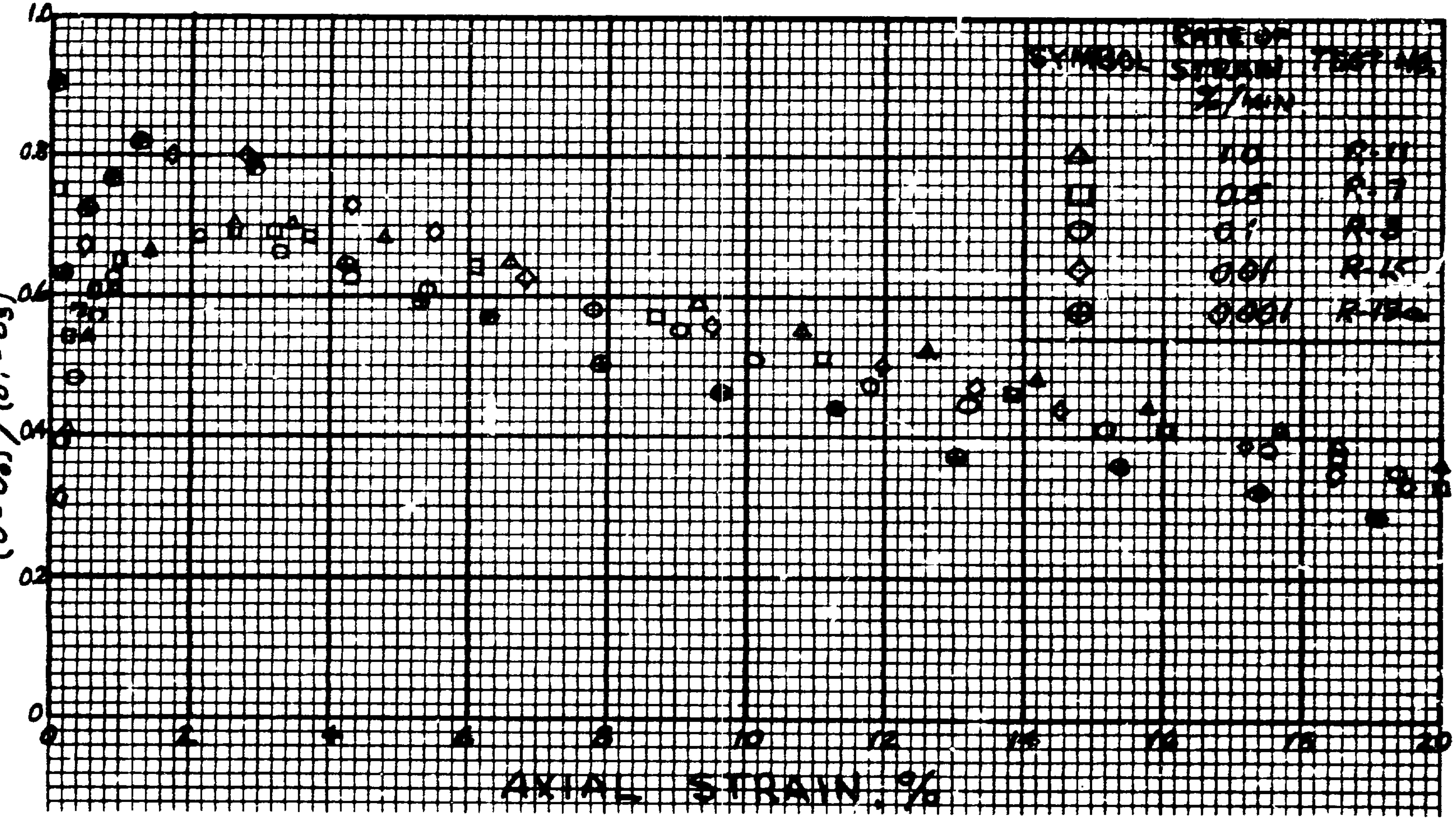



Fig. 17. Pore pressure parameter A versus axial strain.. $\bar{\sigma}_c = 2.93\ kg/cm^2$

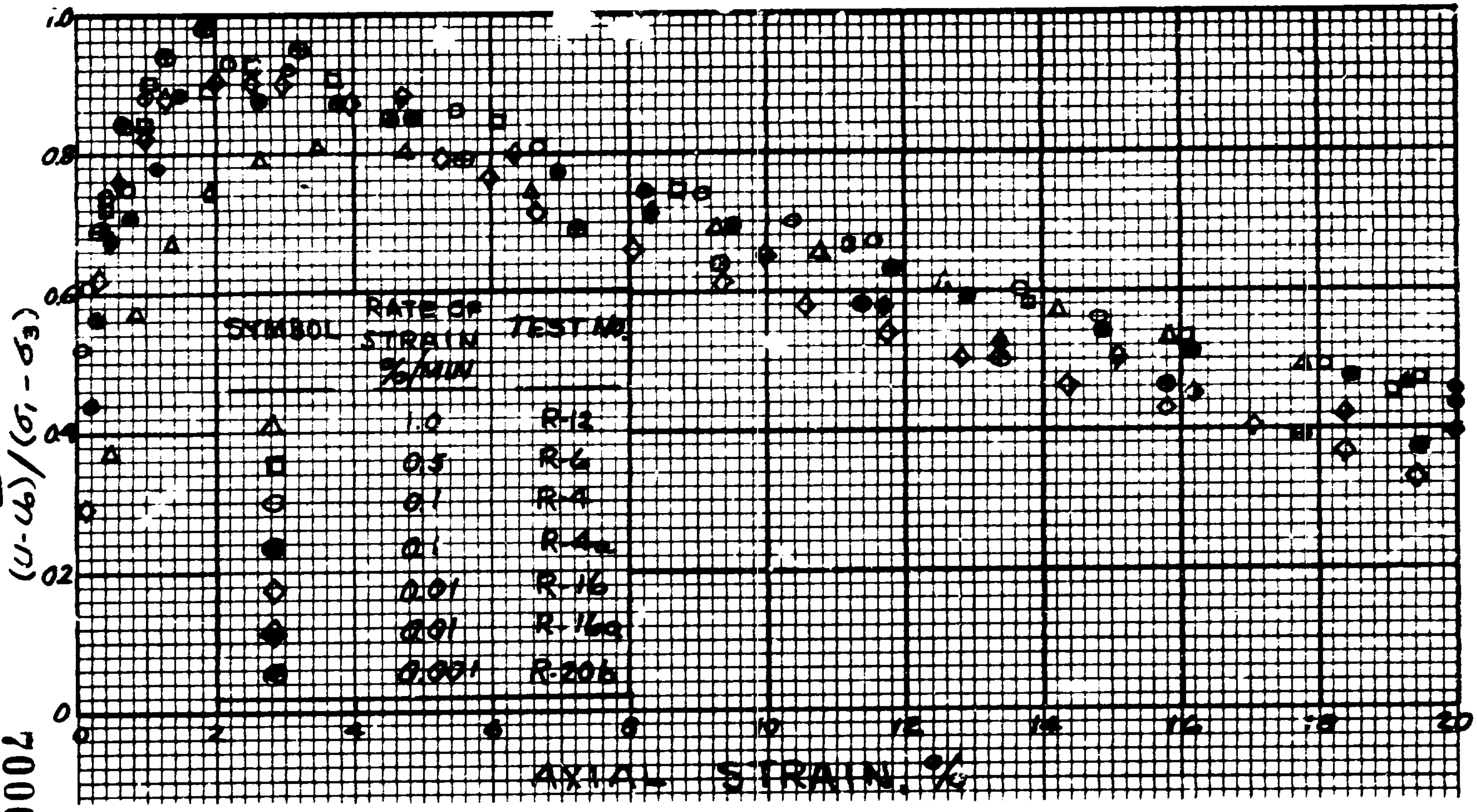



Fig. 18. Pore pressure parameter A versus axial strain. $\bar{\sigma}_c = 4.88\ kg/cm^2$

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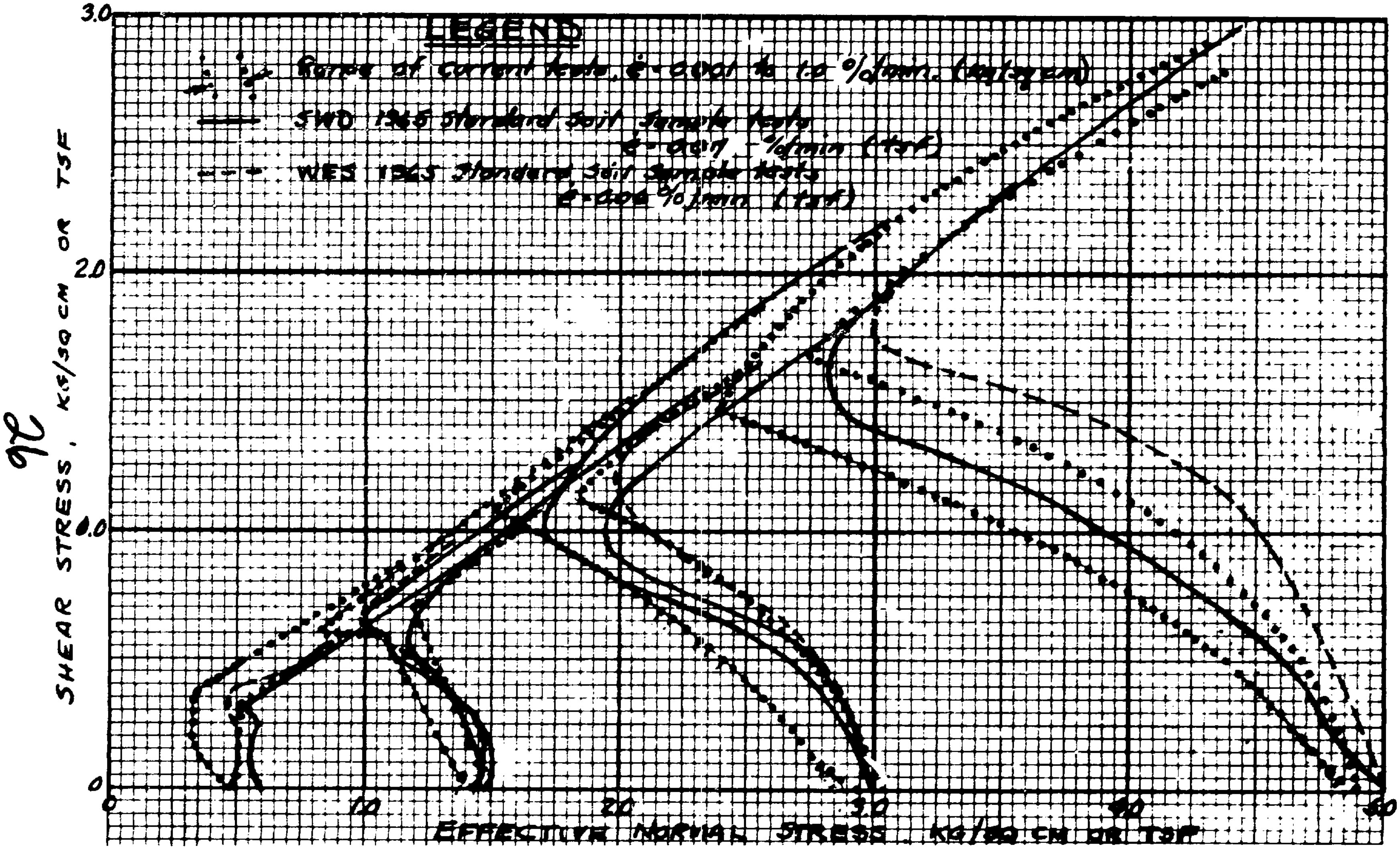


Fig. 19. Stress paths on 60-deg plane

Table 1

**Durations of Various Test Phases**

| Effective Consolidation Pressure $\bar{\sigma}_c$ tsf | Nominal Rate of Axial Strain %/min | Test No. | Durations: From Mixing with Water to Compacting days | Durations: From Compaction to Start of Back-Pressure Saturation hr | Durations: Back-Pressure Saturation* hr | Durations: Consolidation hr | Durations: Axial Loading hr | Durations: From Compaction to End of Test hr |
|---|---|---|---|---|---|---|---|---|
| 0.5 | 1 | R9 | 12 | < 1 | 24[d] | 19 | 0.3 | 43 |
| | 0.5 | R5 | 24 | 18 | 6[c] | 137 | 0.8 | 162 |
| | 0.1 | R1 | 21 | 20 | 22[a] | 50 | 3.3 | 95 |
| | 0.01 | R13 | 3 | < 1 | 19[d] | 21 | 35.5 | 76 |
| | 0.01 | R13a | 1 | < 1 | 20[d] | 19 | 33.3 | 72 |
| | 0.001 | R17b | 3 | 24 | 67[e] | 18 | 333.1 | [illegible] |
| 1.5 | 1 | R10 | 5 | < 1 | 21[d] | 21 | 0.3 | 42 |
| | 1 | R10a | 2 | < 1 | 4[d] | 18 | 0.3 | 22 |
| | 0.5 | R6 | 1 | 18 | 5[c] | 66 | 0.8 | 90 |
| | 0.1 | R2 | 22 | 22 | 21[a] | 25 | 3.3 | 71 |
| | 0.01 | R14 | 3 | < 1 | 17[d] | 21 | 33.3 | 74 |
| | 0.001 | R18 | 4 | < 1 | 19[d] | 21 | 333.1 | 373 |
| 3.0 | 1 | R11 | 6 | < 1 | 19[d] | 21 | 0.3 | 40 |
| | 0.5 | R7 | 5 | 18 | 5[c] | 70 | 0.7 | 94 |
| | 0.1 | R3 | 2 | 72 | 21[b] | 44 | 3.2 | 140 |
| | 0.01 | R15 | 1 | < 1 | 20[d] | 29 | 31.1 | 80 |
| | 0.001 | R19a | 2 | 24 | 67[e] | 18 | 333.1 | 442 |
| 5.0 | 1 | R12 | 1 | < 1 | 22[d] | 18 | 0.3 | 40 |
| | 0.5 | R8 | 6 | 18 | 5[c] | 42 | 0.7 | 66 |
| | 0.1 | R4 | 19 | 20 | 4[c] | 19 | 3.2 | 54 |
| | 0.1 | R4a | 7 | < 1 | 19[b] | 21 | 3.0 | 43 |
| | 0.01 | R16 | 1 | < 1 | 20[d] | 27 | 31.1 | 78 |
| | 0.01 | R16a | Unknown | < 1 | 20[d] | 18 | 31.1 | 70 |
| | 0.001 | R20b | 3 | < 1 | 43[e] | 18 | 333.1 | 372 |

* CP = chamber pressure; BP = back pressure; $\Delta$ = increments.
In all tests, difference between CP and BP was 2 psi.

a <u>R1 and R2</u> - initial CP = 7 and 12 psi, respectively; $\Delta$CP = 5 and 10 psi; CP of 102 and 72 psi, respectively, maintained overnight.

b <u>R3 and R4a</u> - initial CP = 12 and 22 psi, respectively, maintained overnight; $\Delta$CP = 10 or 20 psi.

c <u>R4 through</u> R8 - initial CP varied from 7 to 22 psi; $\Delta$CP = 5 to 20 psi; no increments maintained overnight.

d <u>R9 through R18 (excluding R17b)</u> - initial CP of 7 psi maintained overnight; $\Delta$CP = 5 and 10 psi.

e <u>R17b, R19a, and R20b</u> - 2-psi CP with no BP maintained overnight; then 7-psi CP and 5-psi BP maintained 40 to 54 hr; $\Delta$CP = 5 and 10 psi.

Table 2

Summary of R Triaxial Compression Test Data on Lean Clay (CL)

| Effective Consolidation Pressure $\bar{\sigma}_c$ tsf | Nominal Rate of Axial Strain %/min | Test No. | Initial Conditions $w_o$ % | Initial Conditions $\gamma_{do}$ pcf | Initial Conditions $e_c$ | Initial Conditions $S_c$ % | After Back-Pressure Saturation and Consolidation $u_o$ kg/cm$^2$ | $\bar{\sigma}_c$ kg/cm$^2$ | $w_c$* % | $\gamma_{dc}$** pcf | $e_c$ | $S_c$ % | Actual Rate of Axial Strain %/min | Axial Loading Data: At 15% Axial Strain $u - u_o$ kg/cm$^2$ | $\bar{\sigma}_3$ kg/cm$^2$ | $\sigma_1 - \sigma_3$ kg/cm$^2$ | $\frac{\sigma_1 - \sigma_3}{2\bar{\sigma}_c}$ | $\frac{u - u_o}{\sigma_1 - \sigma_3}$ | At 20% Axial Strain† $u - u_o$ kg/cm$^2$ | $\bar{\sigma}_3$ kg/cm$^2$ | $\sigma_1 - \sigma_3$ kg/cm$^2$ | $\frac{\sigma_1 - \sigma_3}{2\bar{\sigma}_c}$ | $\frac{u - u_o}{\sigma_1 - \sigma_3}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.5 | 1 | R9 | 18.5 | 103.7 | 0.62 | 80.5 | 9.14 | 0.49 | 22.9 | 104.3 | 0.61 | 101.1 | 1.000 | -0.04 | 0.53 | 1.50 | 1.53 | -0.03 | -0.10 | 0.59 | 1.63 | 1.66 | -0.06 |
| | 0.5 | 55 | 18.[illegible] | 103.9 | 0.62 | 79.4 | 8.44 | | 21.9 | 104.2 | 0.61 | 98.5 | 0.440 | -0.02 | 0.51 | 1.42 | 1.45 | -0.02 | -0.06 | 0.57 | 1.57 | 1.60 | -0.05 |
| | 0.1 | R1 | 18.2 | 104.0 | 0.62 | 79.4 | 8.44 | | 21.8 | 104.3 | 0.61 | 98.0 | 0.103 | 0.02 | 0.47 | 1.48 | 1.51 | 0.01 | -0.03 | 0.58 | 1.64 | 1.67 | -0.05 |
| | 0.01 | R13 | 18.8 | 103.7 | 0.62 | 81.5 | 9.14 | | 22.4 | 104.2 | 0.61 | 98.5 | 0.009 | 0.01 | 0.48 | 1.70 | 1.73 | -0.11 | -0.32 | 0.61 | 1.97 | 2.01 | -0.16 |
| | 0.01 | R13a | [illegible] | 103.8 | 0.61 | [illegible].2 | 9.14 | | 22.0 | 104.1 | 0.61 | 100.0 | 0.010 | -0.07 | 0.56 | 1.56 | [illegible] | -0.05 | -0.21 | 0.70 | 1.67 | 1.70 | -0.13 |
| | 0.001 | R17b | 18.[illegible] | 103.[illegible] | 0.61 | 82.8 | 9.14 | | 23.2 | 103.8 | 0.61 | 100.0 | 0.001 | -0.14 | 0.63 | 1.90 | 1.94 | -0.07 | -0.21 | 0.70 | 2.14 | 2.18 | -0.98 |
| 1.5 | 1 | R10 | 18.5 | 103.[illegible] | 0.62 | 80.6 | 9.14 | 1.46 | 21.3 | 106.2 | 0.58 | 99.0 | 1.000 | 0.38 | 1.08 | 3.11 | 1.07 | 0.12 | 0.23 | 1.23 | 3.48 | 1.19 | 0.07 |
| | 1 | R10a†† | 18.6 | 103.[illegible] | 0.61 | 81.4 | 9.14 | | 21.9 | 105.5 | 0.58 | 100.0 | 1.000 | 0.74 | 0.73 | 2.11 | 0.72 | 0.35 | 0.60 | 0.86 | 2.35 | 0.81 | 0.25 |
| | 0.5 | R6 | 19.0 | 103.5 | 0.62 | 82.1 | 8.44 | | 22.8 | 105.3 | 0.60 | 102.2 | 0.440 | 0.59 | 0.87 | 2.30 | 0.82 | 0.23 | 0.46 | 1.00 | 2.66 | 0.91 | 0.17 |
| | 0.1 | R2 | 18.4 | 103.6 | 0.62 | 80.3 | 7.71 | | 22.6 | 106.2 | 0.58 | 104.9 | 0.103 | 0.51 | 0.95 | 2.34 | 0.80 | 0.22 | 0.35 | 1.11 | 2.66 | 0.91 | 0.13 |
| | 0.01 | R14 | 18.9 | 103.7 | 0.62 | 81.9 | 9.14 | | 21.9 | 105.7 | 0.59 | 100.0 | 0.009 | 0.58 | 0.88 | 2.23 | 0.76 | 0.26 | 0.42 | 1.04 | 2.55 | 0.87 | 0.16 |
| | 0.001 | R18 | 18.5 | 103.8 | 0.62 | 90.3 | 9.14 | | 22.0 | 105.6 | 0.59 | 101.0 | 0.001 | 0.30 | 1.14 | 2.74 | 0.95 | 0.11 | 0.12 | 1.34 | 3.05 | 1.04 | 0.04 |
| [illegible] | 1 | R11 | 18.[illegible] | 103.[illegible] | 0.62 | 79.8 | 9.14 | 2.93 | 20.9 | 107.3 | 0.56 | 100.3 | 1.000 | 1.68 | 1.25 | 3.64 | 0.62 | 0.46 | 1.44 | 1.45 | 4.08 | 0.70 | 0.36 |
| | 0.5 | R7 | 18.5 | 103.6 | 0.62 | 80.3 | 7.71 | | 21.2 | 106.9 | 0.57 | 100.0 | 0.480 | 1.58 | 1.35 | 3.63 | 0.62 | 0.43 | 1.55 | 1.56 | 4.03 | 0.69 | 0.35 |
| | 0.1 | R3 | 18.6 | 103.8 | 0.62 | 80.9 | 9.14 | | 21.[illegible] | 106.8 | 0.57 | 100.5 | 0.105 | 1.49 | 1.44 | 3.59 | 0.61 | 0.41 | 1.29 | 1.64 | 3.96 | 0.67 | 0.33 |
| | 0.01 | R15 | 18.6 | 103.5 | 0.62 | 80.2 | 9.14 | | 21.2 | 107.2 | 0.57 | 100.0 | 0.010 | 1.52 | 1.41 | 3.52 | 0.60 | 0.43 | 1.29 | 1.64 | 3.93 | 0.67 | 0.33 |
| | 0.001 | R19a | 18.7 | 103.9 | 0.61 | 82.5 | [illegible] | | 20.9 | 107.2 | 0.56 | 100.0 | 0.001 | 1.48 | 1.45 | 4.12 | 0.70 | 0.36 | 1.25 | 1.68 | 4.67 | 0.80 | 0.27 |
| 6.0 | 1 | R12 | 18.6 | 103.9 | 0.62 | 81.0 | 9.14 | [illegible].88 | 20.3 | 108.4 | 0.55 | 99.3 | 1.000 | 2.92 | 1.96 | 5.34 | 0.55 | 0.55 | 2.60 | 2.28 | 5.97 | 0.61 | 0.44 |
| | 0.5 | R8 | 18.5 | 103.7 | 0.62 | 80.1 | 9.14 | | 20.3 | 108.4 | 0.55 | 99.3 | 0.480 | 2.83 | 2.05 | 5.13 | 0.53 | 0.55 | 2.53 | 2.35 | 5.90 | 0.60 | 0.43 |
| | 0.1 | R4 | 18.6 | 103.4 | 0.62 | 80.2 | 9.14 | | 20.7 | 108.0 | 0.56 | 99.5 | 0.105 | 2.87 | 2.01 | 5.20 | 0.53 | 0.55 | 2.57 | 2.31 | 5.74 | 0.58 | 0.45 |
| | 0.1 | R4a | 18.5 | 103.7 | 0.62 | 80.2 | 9.15 | | 20.1 | 108.1 | 0.55 | 98.4 | 0.110 | 2.83 | 2.05 | 5.27 | 0.54 | 0.54 | 2.50 | 2.38 | 5.86 | 0.60 | 0.43 |
| | 0.01 | R16 | 18.6 | 104.0 | 0.62 | 81.1 | 9.14 | | 20.0 | 109.4 | 0.54 | 99.6 | 0.010 | 2.56 | 2.32 | 5.72 | 0.59 | 0.44 | 2.17 | 2.71 | 6.54 | 0.67 | 0.33 |
| | 0.01 | R16a | 18.6 | 103.5 | 0.62 | 79.9 | 9.14 | | 20.0 | 108.8 | 0.54 | 100.0 | 0.010 | 2.74 | 2.14 | 5.46 | 0.56 | 0.50 | 2.39 | 2.49 | 6.20 | 0.63 | 0.39 |
| | 0.001 | R20b | 18.9 | 103.5 | 0.62 | 82.4 | 9.14 | | 20.1 | 108.3 | 0.54 | 99.3 | 0.001 | 2.67 | 2.21 | 5.66 | 0.58 | 0.47 | 2.33 | 2.55 | 6.51 | 0.61 | 0.36 |

Note: Symbols used in the headings are defined in EM 1110-2-1906.
* Average after-test water content.
** Dry densities listed for R15 and R16 are at water contents of 19.5 and 19.4 percent, respectively.
† Based on volume change indicated by burette during saturation and consolidation.
†† Repeat of test R10 in which leakage was noted in outside connections.

Table 3

Comparison of Water Contents Computed for After-Consolidation Condition and Those Determined After Shear

| Confining Pressure $\bar{\sigma}_c$ tsf | Nominal Rate of Axial Strain %/min | Test No. | Water Contents Computed After Consolidation* $w_c$, % | Average Water Contents at End of Test, $w_f$, % Total Specimen (7 Slices) | Middle 80% (Middle 5 Slices) | Middle 50% (Middle 3 Slices) |
|---|---|---|---|---|---|---|
| 0.5 | 1 | R9 | 22.70 | 22.88 | 22.92 | 23.09 |
| | 0.5 | R5 | 22.83 | 21.94 | 22.01 | 22.20 |
| | 0.1 | R1 | 23.01 | 21.75 | 21.87 | 22.17 |
| | 0.01 | R13 | 22.90 | 22.35 | 22.48 | 22.82 |
| | 0.01 | R13a | 22.97 | 22.63 | 22.75 | 23.00 |
| | 0.001 | R17b | 23.03 | 23.25 | 23.15 | 23.41 |
| 1.5 | 1 | R10 | 21.94 | 21.28 | 21.30 | 21.42 |
| | 1 | R10a | 21.93 | 21.87 | 21.94 | 21.98 |
| | 0.5 | R6 | 22.54 | 22.79 | 22.78 | 23.03 |
| | 0.1 | R2 | 22.07 | 22.61 | 22.67 | 22.99 |
| | 0.01 | R14 | 21.93 | 21.89 | 22.02 | 22.24 |
| | 0.001 | R18 | 22.03 | 22.04 | 22.07 | 22.27 |
| 3.0 | 1 | R11 | 20.97 | 20.86 | 20.89 | 21.04 |
| | 0.5 | R7 | 21.07 | 21.17 | 21.20 | 21.30 |
| | 0.1 | R3 | 21.14 | 21.31 | 21.35 | 21.54 |
| | 0.01 | R15 | 21.25 | 21.21 | 21.28 | 21.42 |
| | 0.001 | R19a | 20.75 | 20.91 | 21.19 | 21.10 |
| 5.0 | 1 | R12 | 20.46 | 20.29 | 20.32 | 20.44 |
| | 0.5 | R8 | 20.29 | 20.26 | 20.30 | 20.44 |
| | 0.1 | R4 | 20.69 | 20.65 | 20.68 | 20.83 |
| | 0.1 | R4a | 20.23 | 20.12 | 20.19 | 20.37 |
| | 0.01 | R16 | 20.29 | 19.95 | 19.97 | 20.15 |
| | 0.01 | R16a | 20.16 | 20.01 | 20.16 | 20.34 |
| | 0.001 | R20b | 20.31 | 20.09 | 20.20 | 20.38 |

* The water content of each specimen after consolidation was computed as follows:

$$w_c = \frac{W_{wo} + (\Delta V_{ws} - \Delta V_{wc})\gamma_w}{W_s} \times 100$$

where

$w_c$ = water content in percent dry weight at end of consolidation

$W_{wo}$ = initial weight of water

$\Delta V_{ws}$ = change in volume of water during saturation as indicated by burette

$\Delta V_{wc}$ = change in volume of water during consolidation as indicated by burette

$\gamma_w$ = unit weight of water

$W_s$ = weight of dry soil

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13. ABSTRACT

The results of consolidated-undrained (termed R test in Corps of Engineers nomenclature) triaxial compression tests with pore pressure measurements performed on Vicksburg silty clay (CL) are presented and analyzed in this report. All triaxial specimens were compacted with a Harvard miniature compactor to 95 percent of standard maximum density with water contents 2 percentage points wet of standard optimum. After back-pressure saturation and consolidation under four different chamber pressures, the specimens were axially loaded at rates of strain varying from 0.001 to 1.0 percent/min. The purpose of the tests was to evaluate the effects, if any, of different rates of strain on the shear strength and deformation characteristics of this particular soil. Data presented include pore pressure observations, magnitudes of deviator stresses, Mohr's diagrams, and stress path plots. R triaxial test results indicate that this lean clay, which has a liquid limit of 34, plastic limit of 22, and plasticity index of 12, is relatively insensitive to the rates of strain used in axial loading. When other materials have been tested at different rates of strain in succeeding phases of the program, more definitive guidance on rates of strain for various fine-grained soils should be possible.

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| Consolidated-undrained tests | | | | | | |
| Pore pressure measurement | | | | | | |
| Soil tests | | | | | | |
| Triaxial compression tests | | | | | | |
| Vicksburg silty clay | | | | | | |